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DISCUSSION OF
THE PROBLEM OF WAVE ACTION
ON EARTH SLOPES
(*Published in November, 1950*)

By Robert Y. Hudson, Sr., Fred C. Walker, Henry H.
Jewell, C. L. Bretschneider and R. R. Putz,
and Martin A. Mason

SOIL MECHANICS DIVISION

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DISCUSSION

ROBERT Y. HUDSON,⁶ M. ASCE.—The use of the Iribarren formula for the design of rubble-mound structures to insure the stability of these structures during wave attack is suggested in this paper. The writer concurs in this recommendation.

The factor γ in equation $\left(W = \frac{K h^3 \gamma}{(\cos \alpha - \sin \alpha)^3 (\gamma - 1)^3} \right)$ is worthy of further consideration, however. Although γ was defined as specific weight in tons per cubic meter, it would have been better, perhaps, to have stated that γ was mass density. Metric tons per cubic meter reduces to grams per cubic centimeter, which is mass density in the centimeter-gram-second system of units. In this system, specific weight has the dimensions of dynes per cubic centimeter. Thus, in the centimeter-gram-second system, the density of fresh water is approximately one, whereas the specific weight of fresh water is about 980. In the American system of units, the density of fresh water at 60° F is 1.938 slugs per cu ft, and the specific weight is 62.4 lb per cu ft.

The Iribarren formula is not dimensionally homogeneous, and this fact precludes its general verification by experimental tests. If the formula is derived in a manner similar to that originally employed, using the same assumptions and force diagram as Mr. Iribarren used in his derivation, the following more complete forms of the equation can be obtained:

$$W = \frac{K' \gamma_f^3 \gamma_r \mu^3 h^3}{(\mu \cos \alpha - \sin \alpha)^3 (\gamma_r - \gamma_f)^3} \dots \dots \dots (2a)$$

and

$$W = \frac{K' \gamma_w S_f^3 S_r \mu^3 h^3}{(\mu \cos \alpha - \sin \alpha)^3 (S_r - S_f)^3} \dots \dots \dots (2b)$$

or if $S_f = 1.00$

$$W = \frac{K' \gamma_w S_r \mu^3 h^3}{(\mu \cos \alpha - \sin \alpha)^3 (S_r - 1)^3} \dots \dots \dots (2c)$$

In Eqs. 2, W is the weight of individual cap rock; K' is an undetermined dimensionless coefficient; γ_w is the specific weight of fresh water; γ_f is the specific weight ($\gamma_w S_f$) of the fluid in which the cap rock is submerged; γ_r is the specific weight ($\gamma_w S_r$) of the cap rock; S_f is the specific gravity (γ_f/γ_w) of the fluid in which the cap rock, is submerged; S_r is the specific gravity (γ_r/γ_w) of the cap rock; μ is the effective coefficient of friction, rock on rock; h is the height of wave at the position of the structure before the structure was constructed (selected design wave); and α is the angle of the seaside slope of the rubble mound, measured from the horizontal.

Eqs. 2 are dimensionally homogeneous and are valid in any consistent system of units. By comparing Eq. 1 with Eqs. 2 or by dimensional reasoning, it can be shown that $K = \gamma_w K'$. Therefore, the values of K' corresponding to $K = 15$ and $K = 19$ are $K' = 0.015$ and $K' = 0.019$, respectively. These

NOTE.—This paper by Martin A. Mason was published in November, 1950, as *Proceedings-Separate No. 44*. The numbering of footnotes and equations in this Separate is a continuation of the consecutive numbering used in the original paper.

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K -values were determined by Mr. Iribarren from observations of a limited number of full-scale structures after wave attack. When the difficulty of obtaining accurate measurements of the seaside slope of rubble mounds after construction is taken into consideration, as well as the difficulty of determining the height of the largest wave that did not displace cap rock from the slope of the structure, it can be seen that the values of K as recommended by Mr. Iribarren could be in serious error. Data obtained from small-scale experiments at the Waterways Experiment Station, Vicksburg, Miss., show that K' is a function of the ratio of water depth at the structure to wave length (d/λ) and the slope of the rubble mound (α). Also, K' is believed to be a function of the ratio of wave height to wave length (h/λ), the percentage of voids in the rubble mound, the shape factor of the rock, and the width of the structure near the still-water level. The importance of these variables, as they affect the design of rubble-mound breakwaters is being studied at the Waterways Experiment Station at the present time. The results of tests completed to date indicate that the breakwater slope (α) is the most important variable contained in K' . It has been found that the average value of K' is about 0.005 for a slope of 1 on 1 and about 0.03 for a slope of 1 on 3. Also, K' tends to increase with decreasing values of the ratio d/λ . Mr. Iribarren assumed a value of unity for the coefficient of friction ($\mu = 1$). If this were true, a rubble mound with a slope of 1 on 1 would fail when attacked by a wave of infinitesimal height. Actually, rubble mounds of 1 on 1 slope will withstand the attack of waves of finite height. The effective coefficient of friction of the rubble used in the small-scale tests at the Waterways Experiment station varied from about 1.01 to 1.10.

Until more accurate and more detailed data are available, it is recommended that the more complete and dimensionally homogeneous form of Iribarren's formula (Eqs. 2) be adopted for design of rubble-mound breakwaters and slope-protection cover, and that values of 0.02 for K' and 1.05 for μ be used for seaside slopes ranging in slope from 1 on 1- $\frac{1}{2}$ to 1 on 3.

FRED C. WALKER,⁷ M. ASCE.—The engineer faced with the problem of designing protective surfaces for earth dams is equally as concerned about the deterioration caused by wave action as he is about the forceful displacement of the protective cover by maximum waves. The continuous action of wetting and drying and freezing and thawing, the continuous tapping produced by wave action, and the grinding produced by sand in the wave-borne water will, if given sufficient time, tear down the strongest protective cover. This is the day to day action of the waves in which magnitude of the waves is relatively unimportant as compared to durability of the protective cover material. It is not difficult to visualize a protective cover designed and constructed to resist maximum wave action according to the most scientific studies, only to find that sufficient deterioration to render the cover completely ineffective has occurred long before the maximum wave action is experienced. This action will, of course, be very different for smooth-surfaced covers than it will be for riprap, just as the effect of maximum waves is different.

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In the field in which rock riprap is considered, a very real need exists for a method of effectively evaluating the relative durability of different kinds of rock. When rock riprap and other types of protective cover are potentially competitive, a method by which such surfaces may be precisely compared is essential. At present (1951) the means of solving problems of this nature is very inadequate, and the experience and judgment of the designing engineer must be relied upon far more than is desirable.

Since 1945 engineers of the Bureau of Reclamation (USBR) of the United States Department of the Interior have been faced increasingly with the necessity of choosing between a somewhat questionable rock found close at hand or a better, more costly rock from a distant source. In some instances the thickness of the protective layer has been increased, in some replacement has been anticipated, and in others top quality material has been used at considerable expense. There have been instances in which larger-sized rock than would be required by any known formula has been specified, not because the adequacy of the formula was questioned but rather because breakdown of the rock by weathering was anticipated. Such factors must be considered both in the creation of a design and in the development of experience data.

As the cost of providing adequate rock-riprap slope protection rises, there is always increased pressure for the adoption of some alternative method of protection in spite of the comparatively poor showing of such methods in the past. Anticipating a vital need for such knowledge, the USBR in March, 1951, let a contract for the construction of a test section in which several design variations, using either asphalt or portland cement, will be subjected to the deteriorating processes of nature. The test section is located on the south side of Bonny Reservoir in eastern Colorado where conditions of greater than average severity are believed to prevail. Local soils or aggregates, comparable to those generally found in the Missouri River Basin, will be used. Although several years are expected to pass before an appraisal can be made of the adequacy of the various methods being tested, studies are in process to determine the nature of the design modifications that will have to be made to the supporting embankment itself. The use of an impervious surfacing and the smooth surface provided by these materials are expected to require extensive modification to present design practices that apply where rock riprap is the slope protection material.

A second element in the design of rock-riprap slope protection that has not received sufficient emphasis concerns both the shape of the stones and size variation in the mass. In the past highly rounded stones have proved to be unsatisfactory on some minor irrigation structures, and their use is now commonly avoided. Flat, slabby rocks are also avoided because of the unsatisfactory surface developed by practical construction methods and the ease with which breakdown is accomplished. Generally, experienced engineers believe that a well-graded rock mass provides better protection against waves than that obtained when a single size of rock is used. Moreover, it will normally be much more economical to provide a graded rock mass than to provide one consisting of a single size. Neither the Iribarren formula nor

the Ishbash formula makes any provision for shape or gradation. The utility of these formulas would be improved if the effect of these factors were evaluated.

Like many of the factors that must be considered in earth dam design, the forces that attack the protective covering are of two different types, both of which have a time relationship. These factors are: (1) the attacking forces whose magnitude increases as the frequency with which they are experienced decreases and (2) the resistive forces that deteriorate at an accelerated rate with the passage of time. The author has described the principal attacking forces and concentrated upon the promotion of a theoretical approach for defense against them. However, the element of judgment based on experience cannot be eliminated. It is equally as dangerous to use the result of a theoretical analysis without reference to experience of the limitations of the theory as it is to use a purely empirical approach.

Within the decade 1940 to 1950 a very pronounced change was introduced in the requirements of many earth dams. The multipurpose reservoir is very much a fact, and carry-over is also an important element in the design of dam structures. The reservoirs of the future will be subjected to a very different relationship between attacking and resistive forces than has been true in the past. Large portions of the embankment surfaces will be either above the water surface or below it for long periods of time. Therefore the laws of probability must be used to establish the chance of finding the reservoir at any given level at the same time as anticipated wave phenomena occur, with the likelihood that sufficient deterioration will have been effected to make the protective cover useless. There is a very real need for the establishment of a guiding principle outlining the degree of risk that may properly be assumed in the design of protective surfacing under such circumstances.

Surprisingly, reservoirs in which protection from wave action is least important are often located where ample supplies of good riprap material are readily available. With modern excavating equipment such material can be placed quite economically, and as a result ample protection is provided in the designs for such structures without much study. In areas in which wave action is severe, the chance that parts of the dam will be threatened is so remote that somewhat questionable devices for wave protection are being used. For example, several dams have been provided with a flattened slope, unprotected by any riprap, on the lower portion of the dam. The same device has been used in cases where low dikes, either in the upper flood-control storage or freeboard area, are required. In some cases, the upper dam sections that are infrequently used for flood control have been protected with inferior rock. In others, size is reduced, and the thickness of the protecting layer is also reduced.

Inland reservoirs may be grouped conveniently into three classes: (a) The small sheltered reservoirs in which the thinnest practical cover would be adequate against any anticipated wave; (b) the normal storage reservoir in which waves between 4 ft and 6 ft high may be expected occasionally; and (c) the very large reservoirs (located principally along the major tributaries of the Mississippi-Missouri River system or the Texas coastal plains) upon which major

wave action will be experienced. The first group of structures obviously will not be greatly benefited by the results of extensive wave study. The second group in which deterioration is a major problem can be treated adequately by existing empirical methods, without appreciable benefits evident from refinements in maximum wave analysis. The third group will benefit very materially from any knowledge developed either in the field of maximum wave action or in the field of durability of protective materials.

HENRY H. JEWELL,⁸ M. ASCE.—A resumé of current knowledge and research on wave action on earth slopes is given in this paper. It also recommends the Iribarren formula for the design of stone protection for earth dams.

Although it may be possible to solve problems of wave characteristics by a rigorous theoretical approach as the author states, the design of protection against wave action on earth dams ought to be accomplished only by the customary combination of theory and practice. This is particularly true of protection by rock riprap.

Some further examination and explanation of the applicability of the Iribarren formula appears necessary. The translation referred to by the author in the section on wave action on dam slopes⁴ shows that the formula was derived for use in the design of rock-fill dikes. Under the heading "Wave Action on Dam Slopes," the author states that "* * * it represents the most logical solution available for designing slope protection cover." If the quoted statement refers to the top layer of rock in an abnormally thick cover of riprap over a filter layer, the formula may be of some value. Alone, it is not likely to offer a practical and economical solution for the design of riprap on earth dams.

Using this formula W is found to be 3,680 lb, assuming the height of wave is equal to 3.5 m (about the maximum for design on the largest reservoirs), the specific weight of stone is 160 lb per cu ft, the slope of the dam is equal to 1 on 2.5, and the value of K is 15. As it would seem that such large stones for a cover layer would be "artificially placed stone," the coefficient K would be 19, and W would then be 4,650 lb. Actual experience with rock riprap has shown that these stones would be heavier than necessary under the conditions stated. Does the author mean that stones of such magnitude should be used for rock riprap on earth dams under the assumption that such protection is the same as rock fill in the formula for design of rock-fill dikes?

Although large stones in the exposed face of the cover are essential, it must be recognized that, if riprap is to be used, the size and weight of stones for protection of earth dams must be within the practical limits of capacity of suitable equipment for quarrying, transporting, and placing the stones. These operations must be done at reasonable cost, under conditions quite different from those governing dike construction in harbors. Although it would be possible to construct riprap of stones weighing 3,000 lb to 5,000 lb, the cost including an adequate filter layer would be prohibitive. Using such

⁸ Chf., Construction Branch, Division of Community Facilities and Special Operations, Housing and Home Finance Agency, Washington, D. C.

large stones for the cover layer would surely require additional quantities of smaller stones both in the top layer and in the underlying filter layer in order to prevent damage caused by washing out the filter layer through the openings between the large cover stones.

Of equal or of greater importance than the weight of stones for earth dam protection is the design of the filter layer that must be placed under the rock cover. The unsuitability of large blocks without a filter layer was shown in the failure of this type of protection on the Belle Fourche Dam many years ago. The importance of a filter layer was emphasized in experience with Kingsley (Nebr.) and Santee-Cooper Dams (S. C.) described in ASCE reports.^{9,10,11}

C. L. BRETSCHNEIDER,¹² JUN. ASCE, AND R. R. PUTZ.¹³—One of the foremost problems concerning wave action on earth slopes is the prediction of the heights of waves in relatively small bodies of water, such as small lakes, bays, and reservoirs. Although a method for forecasting ocean waves has been developed,² little has been done to extend the theory and method to inland lakes and reservoirs. In the Sverdrup-Munk original work on wave forecasting, dimensionless quantities gH/U^2 and C/U versus gF/U^2 were determined for a number of observations at sea, and relationships were also found to exist between gH/U^2 versus gF/U^2 and C/U versus gF/U^2 . In the above, g is the acceleration due to gravity; U is the wind velocity; F is the fetch length, defined as the distance over which the wind is blowing; C is the wave velocity given by $C = gT/2\pi$ for deep water (in which T is the wave period in seconds); and H is the significant wave height, that for statistical purposes is defined as the average height of the highest one-third of all waves present at a given place and time.

It has been a long-standing question whether the same relationships hold for small bodies of water. Observations made at Clear Lake,¹⁴ Calif., and at Abbotts Lagoon,³ Calif., presented much valuable data that were used to extend the dimensionless relationships for relatively small bodies of water. Further studies of wind waves¹⁵ have been made in the wave channel at the University of California, Berkeley, Calif. Additional data have become available from other sources, and it is now possible to revise the Sverdrup-Munk curve, that relates the above dimensionless variables, and to extend the relationship to include all values of gF/U^2 from 0.1 to 200,000.¹⁶

⁹ "Protecting Upstream Slope of Kingsley Dam," by Henry H. Jewell, *Civil Engineering*, Vol. 15, November, 1945, p. 493.

¹⁰ "Rock Riprap Replaces Porous Concrete Slope Protection at Santee-Cooper Project," by Henry H. Jewell, *Civil Engineering*, Vol. 18, January, 1948, p. 14.

¹¹ "Review of Slope Protection Methods," Report of the Subcommittee on Slope Protection of the Committee on Earth Dams of the Soil Mechanics and Foundations Division, *Proceedings*, ASCE, Vol. 74, 1948, p. 845.

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¹³ Mathematician, Inst. of Engineering Research, Univ. of California, Berkeley, Calif.

¹⁴ "The Characteristics of Wind Waves on Lakes and Protected Bays," by J. W. Johnson, *Transactions*, Am. Geophysical Union, Vol. 29, 1948, pp. 671-681.

¹⁵ "An Experimental Investigation of Wind-Generated Waves," by J. W. Johnson and E. K. Rice, *Series 3, Issue 321*, Inst. of Engineering Research, Univ. of California, Berkeley, Calif. (unpublished report).

¹⁶ "Revised Wave-Forecasting Curves and Procedures," by C. L. Bretschneider, *Report No. 155-47*, Inst. of Engineering Research, Univ. of California, Berkeley, Calif., 1951 (unpublished report).

With unlimited wind duration and for values of $g F/U^2$ below 5,000, the relationships for $g H/U^2$ versus $g F/U^2$ and C/U versus $g F/U^2$ on log-log scale paper can be represented conveniently by straight lines, the equations of which are

$$g H/U^2 = 0.0035 (g F/U^2)^{0.423} \dots \dots \dots (3)$$

and

$$C/U = 0.064 (g F/U^2)^{0.313} \dots \dots \dots (4)$$

For $g = 32.16$ ft per sec per sec, $F =$ statute miles, $U =$ miles per hour, $C = g T/2 \pi$, $T =$ seconds, and $H =$ feet, the above equations reduce to

$$H = 0.0313 F^{0.423} U^{1.154} \dots \dots \dots (5)$$

and

$$T = 0.624 F^{0.313} U^{0.374} \dots \dots \dots (6)$$

Another variable, not yet mentioned, that determines the wave height and period is the duration of the wind. For relatively long fetches the duration of

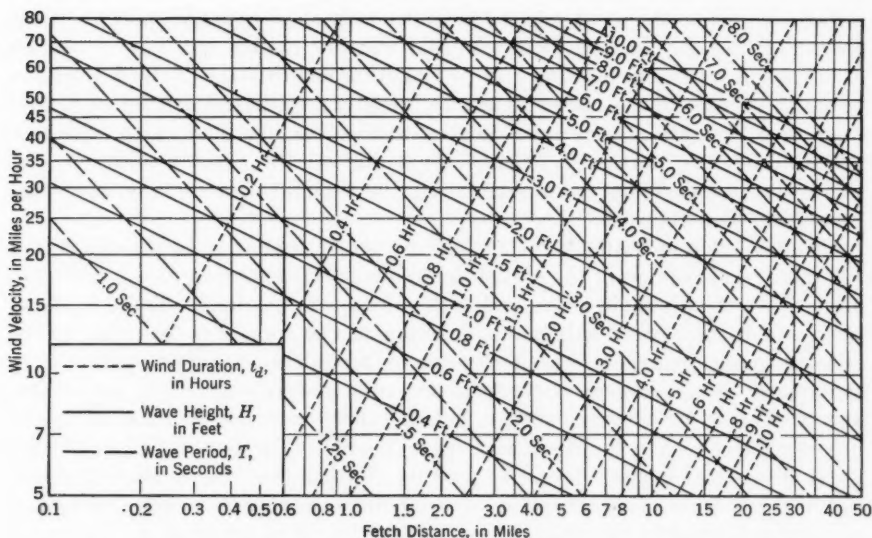


FIG. 1.—WAVE HEIGHT AND PERIOD AS A FUNCTION OF WIND VELOCITY AND FETCH FOR SHORT FETCHES

the wind is the limiting factor, whereas for relatively short fetches, the wind usually is of such a duration that the fetch is the limiting condition. For any fetch length and for a certain wind velocity the wave height and period increase with time until a certain duration is reached; then no matter how long the wind continues to blow, the wave height and period will remain essentially constant; that is, the waves have reached a stable condition. This duration of the wind is called "minimum duration." For waves to reach stable conditions the relationship between wind velocity (U), duration of wind (t_d), and fetch length (F) has the form of

$$t_d U/F = 30 (U^2/g F)^{0.222} \dots \dots \dots (7)$$

that for t_d = hours, F = statute miles, and U = miles per hour reduces to

$$t_d = 2.45 F^{0.778} / U^{0.556} \dots \dots \dots (8)$$

The solutions of Eqs. 4, 6, and 8 are given in Fig. 1. Knowing the wind velocity in miles per hour, the fetch length in miles, and the duration of the wind, one enters the right of Fig. 1, proceeds until either the duration or the fetch length is reached first, and reads off the appropriate wave height and period.

The author has pointed out the necessity of relating the maximum wave height to the significant wave height. Some work on this has been done^{17,18} in the analysis of data from ocean wave recorders. The variability in the heights of consecutive waves from a storm may be examined from several points of view. For example, the heights of waves produced and propagated under completely specified conditions vary with time, if observed at a fixed point, and vary in space, if observed at a fixed time. Moreover, a wave observed at a given time and place, arising from incompletely specified conditions (for instance, a storm for which the wind velocity is known only as an average value or whose characteristics are derived from synoptic weather maps), will vary with the exact nature of the generating and propagating conditions. Thus a weather-based wave forecast may be expected to give only a statistical average of individual wave heights. The wave heights themselves are describable only by probabilities, the particular heights depending on the exact conditions. In this way the individual wave heights may be thought of as having statistical probability distributions.

Studies have indicated a rather high degree of regularity in the shape and the relative dispersion of the statistical frequency distribution of wave heights observed in the ocean, in lakes, and in the laboratory channel. These regularities seem to hold with only minor changes in the constants occurring in the relations describing them, whether the waves are in the area of generation or in the area of decay, in deep water or in the zone of breaking. The regularities also appear to be of value for predicting other statistical features of wave-height patterns when the significant wave height (H) is known.

A theoretical model,¹⁹ found to provide a satisfactory fit to data that consist of the heights of successive waves occurring over a given length of time at a given point, involves two parameters. These are, first, the relative dispersion, measured by the ratio (σ/μ) of the standard deviation to the arithmetic mean of the set of successive wave heights and, second, the degree of asymmetry, measured by the skewness coefficient (α_3) for the frequency distribution of the consecutive wave heights. Observed values of the ratio σ/μ and of the coefficient α_3 show relatively little fluctuation, averaging between +0.3 and +0.7 and between +0.0 and +0.8, respectively, for various groups of data. Typical

¹⁷ "An Analysis of Data from Wave Recorders on the Pacific Coast of the United States," by R. L. Wiegel, *Transactions, Am. Geophysical Union*, Vol. 30, October, 1949, pp. 700-704.

¹⁸ "Results on Research on Surface Waves of the Western North Atlantic," by H. R. Seiwel, *Papers, Physical Oceanography and Meteorology*, Massachusetts Inst. of Technology, Cambridge, Mass., and Woods Hole Oceanographic Inst., Woods Hole, Mass., Vol. 10, 1948, p. 56.

¹⁹ "Wave Height Variability; Prediction of the Distribution Function," by R. R. Putz, Series 3, *Issue 318*, Inst. of Engineering Research, Univ. of California, Berkeley, Calif., December, 1950 (unpublished report).

values of these parameters for wind waves on Abbotts Lagoon³ were found to be $\sigma/\mu = 0.585$ and $\alpha_3 = 0.6$. Based on these values, the family of curves in Fig. 2 shows the proportion of waves that may be expected to be above any given height when the significant height (H) is known, the wave system being assumed to be in a steady state. The letter symbols used in Fig. 2 can be defined as follows: t_d = the duration of the storm, in hours; $t = t_d$ minus the build-up time; T = the period of the wave, in seconds; h = the height in feet above which a given percentage of wave heights occur or the height of the long-run-average maximum wave. It is seen from the curves that 13.0% of the wave heights are above the significant wave height. For example, for a significant

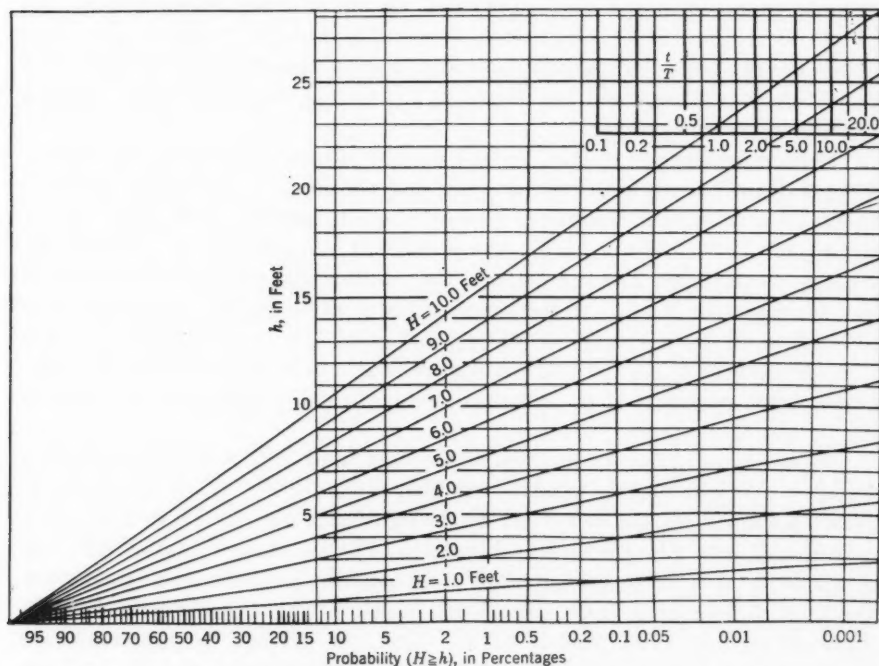


FIG. 2.—WAVE HEIGHT DISTRIBUTION AND LONG-RUN AVERAGE MAXIMUM WAVE HEIGHT

wave height of 3 ft, the corresponding curve yields a value of 0.01% for the long-run expected proportion of waves higher than 7 ft in height. In this case, if waves above 7 ft in height are of interest, such a statement implies the necessity, on the average, of a long wait for waves of such height. During an extended period of time, weather and wave conditions may undergo marked changes, creating a situation significantly different from that upon which the earlier probability of occurrence was based.

If the wave-height system may be assumed to be in a steady state (the distribution of wave heights is independent of the time), Fig. 2 will provide an indication of the maximum wave height to be expected for a system of waves lasting a known length of time (t , measured in hours), and of known average

period (T , measured in seconds), when the significant wave height is known. For example, let $H = 4.0$ ft, $t = 1.0$ hr, and $T = 4.0$ sec; then, referring to the proper H -curve, it is found that the value of h corresponding to the value $t/T = 0.25$ is 8.4 ft. Under these conditions, then, our model gives 8.4 ft as the average value of the maximum wave height. Since the observed maximum value is not determined by the conditions given in such an example, it should be expected to fluctuate about its average value from one storm of this type to the next, being higher, in fact, than the average of its values approximately 45% of the time.

Although much observational work on long sequences of consecutive waves remains to be done, the results obtained may be of value in ascertaining the effects of certain weather conditions. It would be desirable to know much more about the temporal homogeneity of wave processes produced by various wind conditions as well as about the degree and nature of the dependence between successive observed waves.

Further application of Figs. 1 and 2 may be illustrated by using the Santee-Cooper Dam¹⁰ as an example. In this case the fetch is limited to about 15 miles. For a typical condition, assume a wind velocity of 30 miles per hr lasting for 12 hr. From Fig. 1, the significant wave height $H = 5.0$ ft, the average wave period $T = 5.3$ sec, and the duration of wind necessary to reach steady-state $t_d = 3$ hr. At the point of observation for the additional 9 hr ($12 - 3 = 9$), from Fig. 2 for $t/T = 9/5.3 = 1.70$, it is determined that the average value of H_{\max} , the maximum wave height to be expected over the 9-hr. steady-state period, is 12 ft. Of the individual wave heights, 13% are at least 5.0 ft, and 87% are less than 5.0 ft.

When waves approach the face of the dam, they will tend to peak-up and break at a height slightly greater than that given by Fig. 1. For periods, T , between 4 and 6 sec, the waves having the significant wave height in deep water will break²⁰ with a height increased by 10 to 15%, and they will break when the depth of water is 1.3 to 1.5 times the wave height. In the preceding example, the 5.0-ft, 5.3-sec deep-water wave would peak-up to 5.4 ft and break when the depth at normal water level is 6.9 ft. Also, the average maximum breaker height to be expected in the long run would be 13.0 ft (from Fig. 2). In this particular example, since the freeboard is 18 ft, it would be expected that in the long run the average storm (having a wind velocity of 30 miles per hr duration of wind of 12 hr, and fetch length of 15 statute miles) would not generate a wave that would go over the top of the dam. However, for stronger winds and longer durations, greater significant wave heights would be developed, and higher maximum wave heights would be expected.

When waves approach perpendicular to the face of the dam, they will tend to spill or splash the water against the face of the dam, and the returning water under the influence of gravity would cause rip currents. Waves approaching at an angle to the face of the dam would produce a littoral current parallel to the

²⁰ "Breakers and Surf, Principles in Forecasting," Hydrographic Office, U. S. Navy Dept., Publication No. 234, Washington, D. C., November, 1949.

face of the dam and also a return rip current. A theoretical relationship for the littoral current velocity as a function of wave height, period, beach slope, and the angle of approach of the wave can be applied to field and laboratory observations.²¹ The investigation that established this relationship covered beach slopes between 0.016 and 0.26. The slope of the Santee North Dam (North Carolina) is about 0.36, and the waves may behave somewhat differently. The waves would tend to splash or slap against the face of the dam before or at the point of breaking, and their damaging effect would be due both to the momentum of the water and the return current. The impact and the current both depend on the maximum wave height, period, and the slope of the face of the dam. The effect of the riprapping in place of porous concrete for the upstream face would be to diminish the above forces.

It should be noted that the equations for the wind-generated waves can be used for larger fetches than are represented in Fig. 1, provided they do not exceed $gF/U^2 = 5,000$. Since gF/U^2 decreases as the wind velocity squared increases, the limit of the equation is greater for higher wind velocity with increasing fetches. Of particular significance, Eqs. 4, 6, and 8 are dimensionally correct as seen from Eqs. 3, 5, and 7. Fig. 2, with the aid of Fig. 1, can be used to advantage for any particular storm to indicate the maximum wave height to be expected. Although more information on the behavior of waves for particular cases is necessary, the above discussion, with the aid of the references given, completes certain aspects of the study relating to wind-generated waves.

MARTIN A. MASON,²² Assoc. M. ASCE.—The purpose of the paper was to present a resumé of current knowledge and research (as known to the writer) on wave action on earth slopes. It was hoped that, by discussion, the profession would supply deficiencies attributable to the writer's shortcomings and contribute additional knowledge or hypotheses pertinent to the problem. In this respect the results obtained are perhaps somewhat disappointing. The four discussions contributed are valuable beyond question. They offer critical comment and contribute useful knowledge on this poorly known subject. However, if the lack of a wider response on the controversial subjects of the paper and on the calculated failure to mention the importance of filter layers may be interpreted as an indication of complacency and satisfaction of the profession with present conditions, then the situation is one which the engineering profession cannot view with pride.

The writer can offer little valuable comment on the excellent discussions so kindly contributed, except to concur in the comments, and to clarify a statement apparently misunderstood by Mr. Jewell. The Iribarren formula is not recommended by the writer for the design of slope protection but is recommended as the most logical available design formula for determining the size of the cover stone of slope protection.

²¹ "The Prediction of Longshore Currents," by J. A. Putnam, W. H. Munk, and M. A. Traylor, *Transactions*, Am. Geophysical Union, Vol. 30, 1949, pp. 337-345.

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This opportunity is taken also to call attention to the May, 1950, publication of a generalization of the Iribarren formula.²³

The additional data on prediction of waves should find much application to reservoir situations, although the problem of determining applicable fetch and wind velocity is vexatious still.

Continued study and research on slope protection are indicated certainly; existing efforts in this field should be encouraged and expanded with vigor.

²³ "Generalization of the Formula for Calculation of Rock Fill Dikes and Verification of its Coefficients," by R. Iribarren Cavanilles and Casto Nogales y Olano (translation), *Bulletin*, Beach Erosion Board, Vol 5, 1951, p. 4.

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